EXHIBIT "C"

REINFORCED CONCRETE WALLS: RECENT RESEARCH & ACI 318-2001

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Abstract

New provisions are currently being developed for Chapter 21 of ACI 318-2001. A major goal of this code writing effort is to unify the seismic design provisions for reinforced concrete walls for the Uniform Building Code, NEHRP Guidelines, ACI 318, and ultimately, the International Building Code. The new provisions will be drafted using existing ACI 318-95 and UBC-97 provisions. In addition, new material may be added to address the design of stout walls and walls with openings. Recent research is reviewed and areas where changes are being considered are discussed. A first draft of the provisions were balloted by Committee 318-H in September 1997. Based on the balloting process, final provisions should be completed for the 2001 ACI code.

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Introduction

Significant emphasis has been placed on developing new, technically sound, design provisions for reinforced concrete walls in the last ten years. This trend developed based on research on the behavior of wall buildings in Chile following the strong earthquake in 1985 (Wood, 1991; Wallace and Moehle, 1992). Wallace and Moehle (1992) identified that, for a majority of cases, the UBC-85 ("Uniform", 1985) provisions were overly conservative for the design of slender walls and that the role of stiffness was not adequately considered in the code. Based on these conclusions, a displacement-based design methodology was developed (Moehle and Wallace, 1989; Wallace and Moehle, 1992; Wallace, 1994) and experimental studies were conducted to validate the approach (Thomsen and Wallace, 1995; Taylor, Cote and Wallace, 1997).

Experimental studies of structural walls with rectangular and T-shaped cross-sections (Thomsen and Wallace; 1995; Taylor and Wallace, 1995) have been conducted to investigate wall behavior, to validate the use of a displacement-based design approach, and to investigate the behavior of slender walls with openings (Fig. 1). The walls were approximately 1/4 scale and cyclic lateral loads were applied at the top of the walls. A constant axial load of approximately $0.10A_g f'_c$ was maintained for the duration of the testing. Test results for four wall specimens are discussed in the following paragraphs. The walls were tested in an upright position under reversed cyclic load.

The wall specimens were designed using flexural strength requirements of the Uniform Building Code ("Uniform", 1994) as shown in Fig. 2; however, transverse reinforcement required at the wall boundaries was evaluated using displacement-based design (Wallace, 1995). Capacity design was used to ensure the location of primary inelastic flexural deformations would occur at the base of the wall and that adequate shear strength was provided. A strut-and-tie model (Fig.

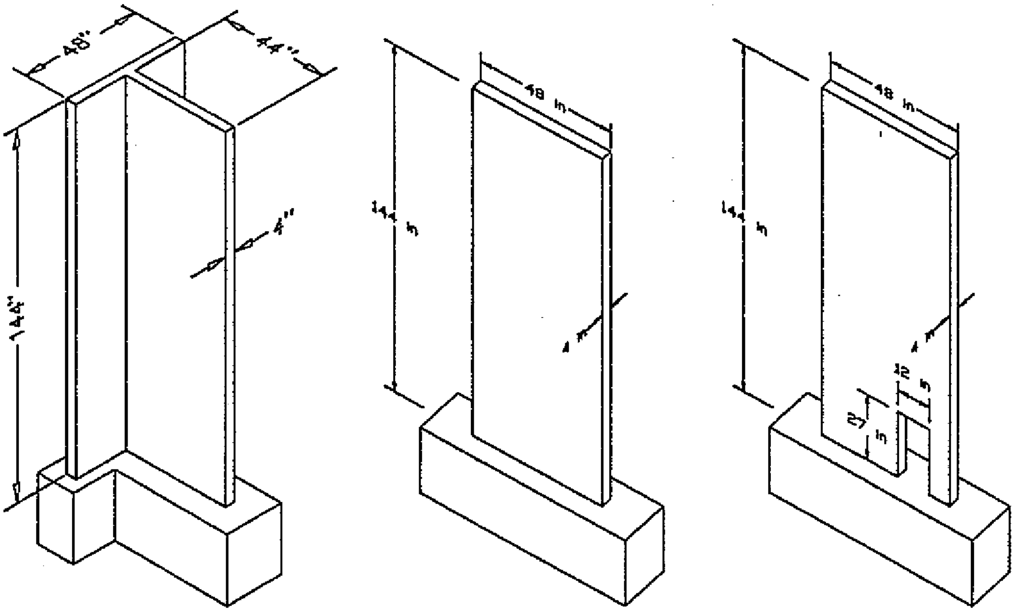
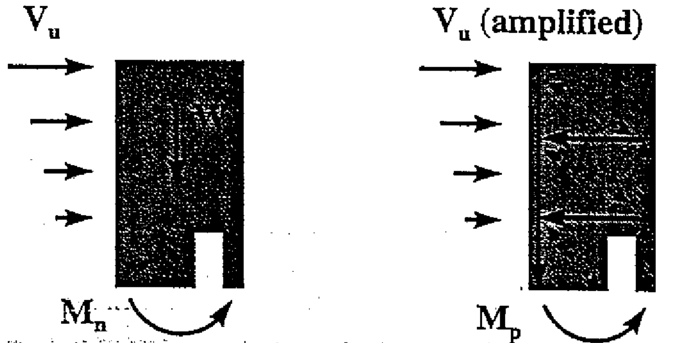


Figure 1 Geometry of RC Wall Specimens



(a) Flexural Strength (b) Shear Strength

Fig. 2 Wall Strength Modeling

2b), with shear based on the probable moment capacity, was used to assess load path and shear strength requirements for the walls with openings (Taylor, Cote, and Wallace, 1997).

Reinforcing details for two of the walls are presented in Figures 3, 4, and 5. Transverse reinforcement at the wall boundaries was selected assuming a design drift level of 1.5% of the wall height using a displacement-based design procedure (Wallace, 1995). The first specimen, RW1, had a rectangular cross-section with eight #3 deformed bars at each wall boundary (Fig. 3, 4). Web reinforcement consisted of deformed #2 bars and transverse boundary reinforcement consisted of 3/16 in. diameter hoops and two cross-ties spaced at 3 inches ($8d_b$, where d_b is the diameter of the boundary vertical reinforcement). Specimen RW2 was identical to RW1, except that hoops spaced at 2 in. were used at the boundary regions (no cross-ties). Specimen TW2 was evaluated based on the expected behavior of the T-shaped wall cross-section (Wallace, 1995). The evaluation for TW2 indicated that more stringent detailing was required at the boundary of the wall web, as well, additional shear reinforcement was needed (Fig. 5). However, details were relaxed at the web-flange intersection and at the flange boundary (note that only uniaxial loads in the plane of the wall web were applied to the specimens). For specimen RW3-O, additional transverse reinforcement was provided at the "boundary column" (hoops and two cross-ties at 2 in. spacing) and additional horizontal and vertical web steel was used over the opening and in the wall panel adjacent to the opening. Additional information on the specimens and the test results are available in the following reports: Thomsen and Wallace (1995) and Taylor and Wallace (1995).

Lateral load versus lateral displacement response at the top of the wall is presented in Fig. 6 for three specimens. Good performance was observed for all specimens. Specimen RW2 (Fig. 6a), was subjected to two cycles of drift at 2.5% (design based on 1.5%) before failing in a combined crushing/buckling mode. Specimen RW1 failed due to buckling of the longitudinal boundary bars on the first cycle to 2.5% drift, which was expected based on the provided spacing of transverse reinforcement ($s = 8d_b$). Figure 6b plots test results for specimens TW2 and indicates very good performance. The wall was subjected to a large number of displacement cycles up to 2.5% lateral drift (4 cycles at 1, 1.5, 2 and 2.5% drift) before failure occurred due to lateral web instability. Even though the flange reinforcement was subjected to significant tensile strains, the experimental results indicate that special detailing requirements for the wall flange and the web-flange intersection are not required. Overall lateral load versus top displacement relations for RW2 and RW3-O (Fig. 6c) are essentially identical, indicating that use of displacement-based design in conjunction with a truss model and capacity design provides a sound method for the evaluation of walls with openings.

A comparison of observed behavior for the three specimens indicates that a displacement-based approach can be used to predict both drift capacities and failure modes of the wall specimens (Thomsen and Wallace, 1995). Analysis of measured responses also indicates that the axial strain distribution at the base of the walls was reasonably linear and that moment-curvature response could be predicted quite accurately using typical models for the reinforcement and the confined and unconfined concrete (Fig. 7 and 8). These results are valuable since they indicate that the assumptions and analytical tools commonly used for the design of slender walls provide reliable estimates of the strength, stiffness, and deformation capacity.

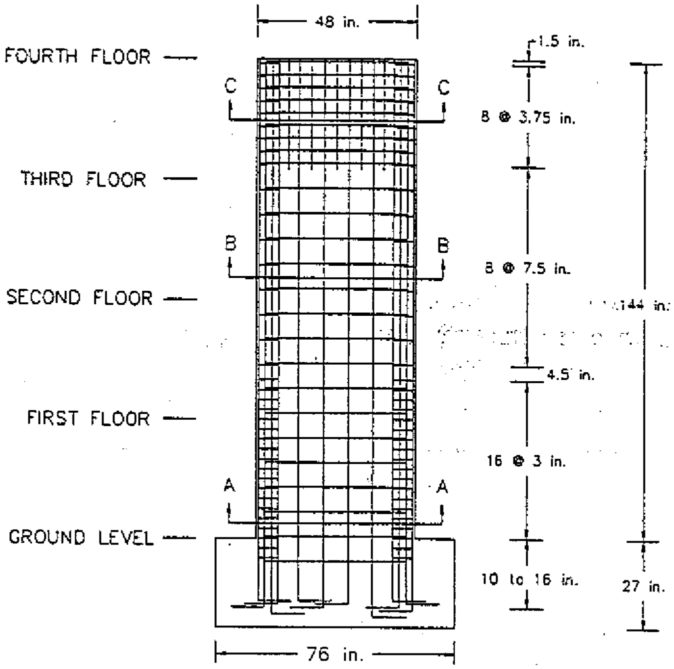


Figure 3 Specimen Details - RW1

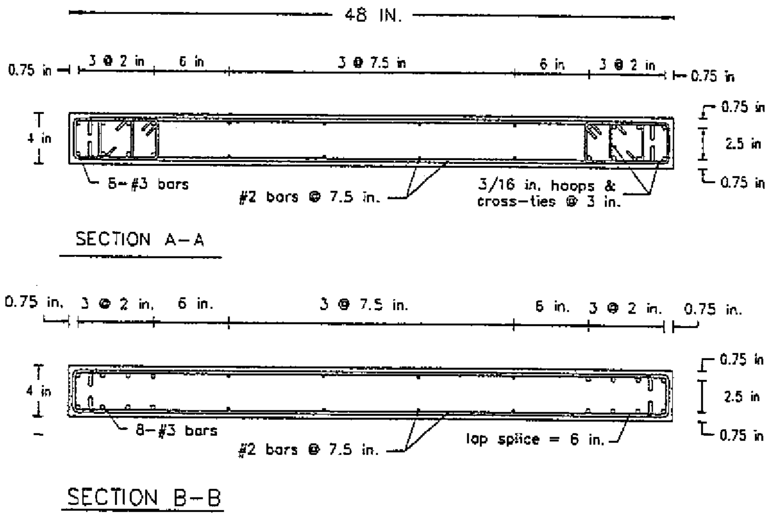


Figure 4 Geometry and Reinforcing Details - RW1

Experimental studies also have been conducted on low-rise walls with openings by Yañez et al. (1991). The walls were 2.3 m tall, 2.0 m long, and 120 mm thick. Six walls were tested, one solid, three with non-aligned openings, and two with aligned openings creating coupling beams. For walls with non-aligned openings, design was based on using strut and tie models in combination with capacity design, since commonly used code equations for solid walls do not provide adequate design guidance. The weak link in the wall resistance was selected to be a main tension tie such that yielding of tension reinforcement would occur prior to crushing of the concrete struts. This process is depicted in Figure 9, which presents a elevation view of a low-rise wall with openings and a strut and tie model superimposed.

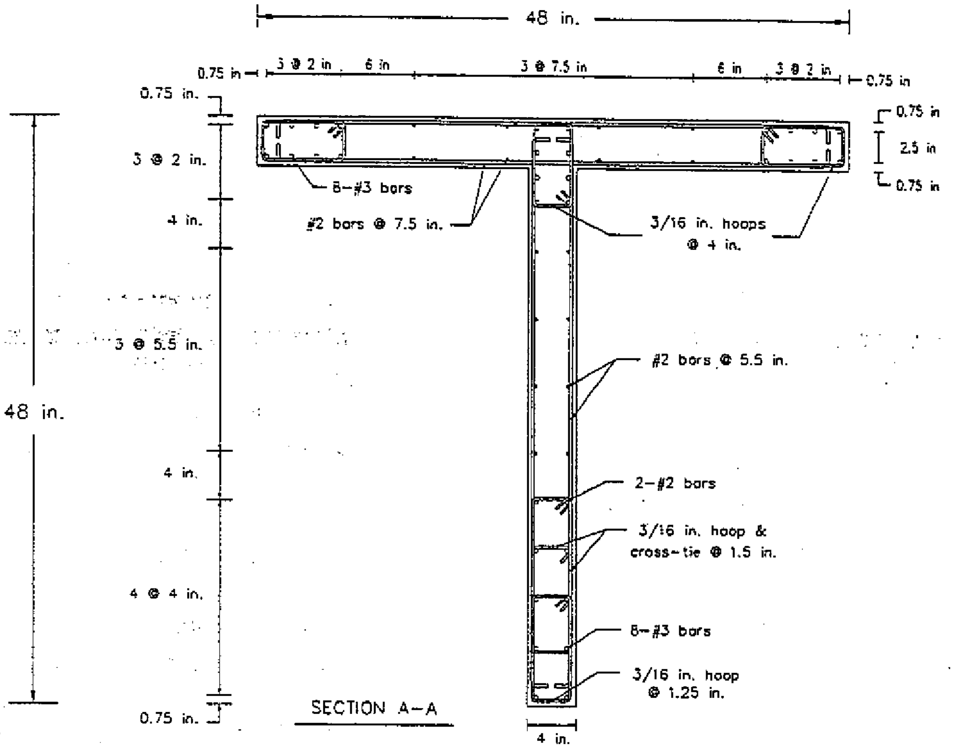


Figure 5 Geometry and Reinforcing Details - TW2

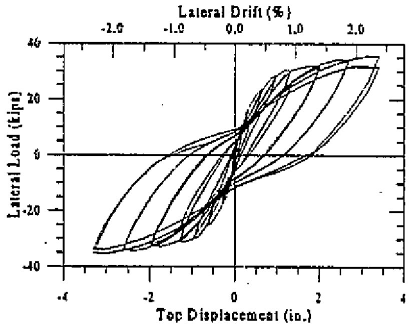


Fig. 6a Specimen RW2

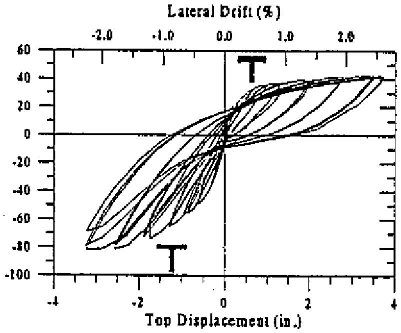


Fig. 6b Specimen TW2

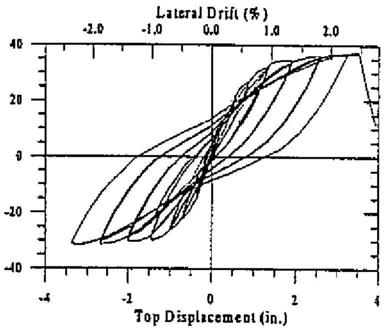


Fig. 6c Specimen RW3-O

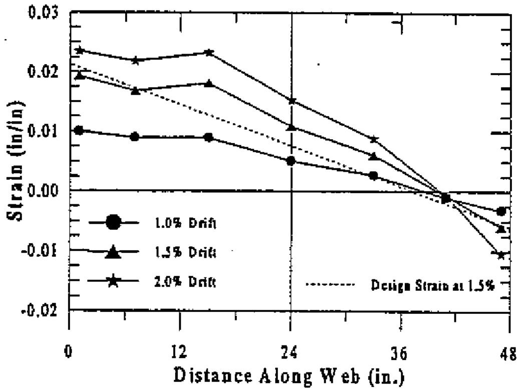


Fig. 7 Strain Profile - RW2

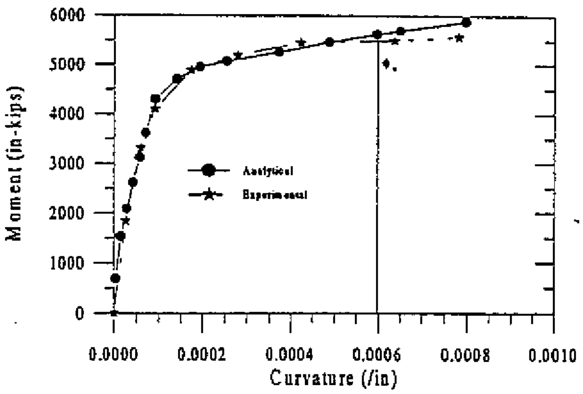


Fig. 8 M-ϕ Response - RW3-O

The walls tested by Yañez et al. (1991) achieved fairly high drift ratios, on the order of 1.5 to 2%, before significant deterioration; therefore, even low-rise walls with tension yielding can undergo substantial yielding. The experimental results indicate use of strut and tie models with capacity design is a reasonable design methodology for a system that would be difficult to evaluate with typical code expressions.

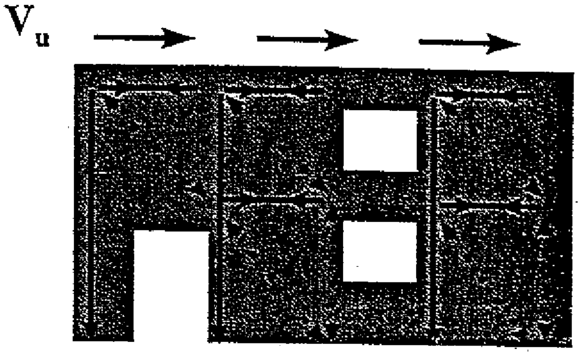


Fig. 9 Strut and Tie Model for Low-Rise Walls

ACI 318-2001: Code Development

A joint ACI-SEAOC task group was formed to identify important aspects of wall design that should be addressed to bring the UBC, ACI, and NEHRP (IBC) provisions into agreement. The ACI 318-95 provisions, although generally quite conservative, have the advantage of being easy to apply, as well, they also cover a broad spectrum of cases (e.g., cantilever walls, walls with interruptions, walls with openings, low-rise walls). The UBC-97 provisions incorporate some of the latest research on displacement-based design; however, the provisions apply primarily to slender cantilever walls. The consensus view was to develop the new ACI provisions by starting with the newer UBC provisions, and update them as needed. At the time of the writing of this paper, a first draft of the ACI 318-2001 provisions are being balloted. The following sections provide background information on the provisions in UBC 1997, as well as present discussion on areas where modifications and additions are being considered for the ACI 318-2001 provisions.

Slender Walls:

New provisions for evaluating detailing requirements for reinforced concrete walls were included in UBC-94 and updated in UBC-97 ("Uniform", 1994, 1997). The provisions, primarily contained in Section 1921.6.5, were developed using a displacement-based approach by the Structural Engineers Association of California and apply to slender walls with constant cross section over the height of the building. Both a simple procedure and a detailed design procedure are provided. The simple procedure is meant to be a rapid screening technique that can be applied to a majority of typical design cases. These procedures are outlined in the following subsections. For some of the provisions, a short discussion is given to provide some history as well as to identify limitations.

Simplified Approach: Rapid Screening: Special detailing requirements at wall boundaries are not required for geometrically symmetrical and unsymmetrical wall cross-sections with axial load less than $0.10A_gf'_c$ and $0.05A_gf'_c$, respectively, which satisfy either of the following conditions:

$$\frac{M_u}{V_u l_w} \le 1.0 \tag{1a}$$

or

$$V_u \le 3\, l_w t_w \sqrt{f'_c}\ \text{psi} \quad \left(0.25\, l_w t_w \sqrt{f'_c}\ \text{MPa}\right) \quad \text{and} \quad \frac{M_u}{V_u l_w} \le 3.0 \qquad (1b)$$

where t_w is the wall thickness, A_g is the gross wall area, f'_c is the concrete strength, V_u is the design wall shear, and M_u is the design wall moment. The second condition on (1b) was added in UBC-97 to address the need to confine slender walls with low shear stress (Wallace, 1996).

Use of rapid screening techniques was reviewed by Wallace (1995, 1996). These studies indicate that the limits used in UBC-94 or UBC-97 may be conservative or unconservative depending on the given design case. To adequately account for the important design variables, the design level of lateral loads (R_w) must also be considered. It is noted that this is difficult to do within the ACI Building Code since ACI does not deal with the loading side of the equation. This issue may be addressed by providing a simple approach that is load independent, as well as including general language about what must be considered if a more detailed approach is used. Some of this material may be placed in the Commentary of ACI 318-2001.

For walls not meeting the conditions of (1), two alternatives are available. The first alternative applies to walls with $P_u < 0.35\ P_0$, where P_0 is the wall nominal axial load strength at zero eccentricity. These walls shall have boundary zones with special transverse reinforcement at each end of the wall a distance varying linearly from $0.25l_w$ to $0.15l_w$ for P_u varying from $0.35P_0$ to $0.15P_0$. The boundary zone shall have a minimum length of $0.15l_w$. The lateral load resistance of walls with $P_u > 0.35P_0$ is neglected; design is governed by requirements for deformation compatibility (UBC, 1994; Section 1631.2.4; NEHRP Sec. 2.2.2.4.3 for UBC-97). This provision is difficult to meet for core walls and changes are being considered that will allow all walls, regardless of axial load level, to be treated the same. The UBC-94 provisions are quite unconservative for most cases since the check on deformation compatibility is based on a deformation multiplier of $3R_w/8$ versus the more realistic multiplier of R_w used in UBC-97 for walls. Also, Wallace (1996) identified that the requirements tend to be unconservative for higher levels of axial load and for flanged walls, which is particularly undesirable given the potential for brittle behavior for these condition.

An alternative approach being considered for ACI 318-2001 would involve defining the effective wall cross section, including any effective flange widths, and then assessing equilibrium requirements to compute the neutral axis depth. Since a P-M interaction diagram will typically be generated to check design strength, this information should be readily available. A simplified approach to detailing the wall boundaries would involve providing special transverse reinforcement over some portion of the compression zone (possibly one-half). The detailed approach, described in the following section, would remain essentially the same, that is, the curvature and strain demands on the wall would be computed to assess detailing requirements. A simplified expression to compute the curvature demand is being considered (Moehle, 1992).

Detailed Approach: Displacement-Based Design: As an alternative to using the rapid screening approach, UBC-97 provides a more detailed evaluation technique for slender, cantilever, walls. The approach is based on using displacement-based design similar to that presented by Wallace and Moehle (1992) and Wallace (1994). This approach is presented and discussed in the following paragraphs.

Use of displacement-based design involves estimating the design displacement (usually the maximum displacement at the top of the wall) in terms of the yield displacement and the inelastic displacement. The displacement at the top of the wall is obtained for the code prescribed lateral forces (reduced by R_w) using a cracked section model with either a fixed or flexible base, or it can be obtained using dynamic analysis procedures. The resulting displacement for the reduced force level is multiplied by $3R_w/8$ in UBC-94 to obtain the design displacement; however, the multiplier was increased to R_w in UBC-97 to provide a more realistic estimate of maximum displacement response (Wallace, 1996). The definition of wall yield curvature was also changed in UBC-97 to $0.003/l_w$ to address an oversight in the UBC-94.

The general process for establishing the contribution of elastic and inelastic wall deformations is depicted in Fig. 10. The model shown in Fig. 10 is for a fixed-base condition; care should be exercised for flexible-base models to ensure that the displacement component due to foundation rotation is not over-estimated (because base rotation may be limited by the wall flexural strength). Combining equilibrium requirements with reasonable assumptions on yield curvature and "plastic hinge length" for the model in Fig. 10, a simple relationship can be derived between roof displacement response and wall strains (Wallace 1994). A similar approach is used in the UBC provisions. Special transverse reinforcement must be provided at the wall boundary where the compression strain exceeds 0.003. The length of the confined boundary region can be assessed using the computed strain distribution; however, it must be greater than a minimum value.

Detailing Requirements: Where special transverse reinforcement is required (for walls where the extreme fiber compression strain exceeds 0.003) all vertical reinforcement in the boundary zone shall be confined by hoops or cross ties with steel area not less than

$$A_{sh} = 0.09\, s h_c \frac{f'_c}{f_{yh}} \tag{2}$$

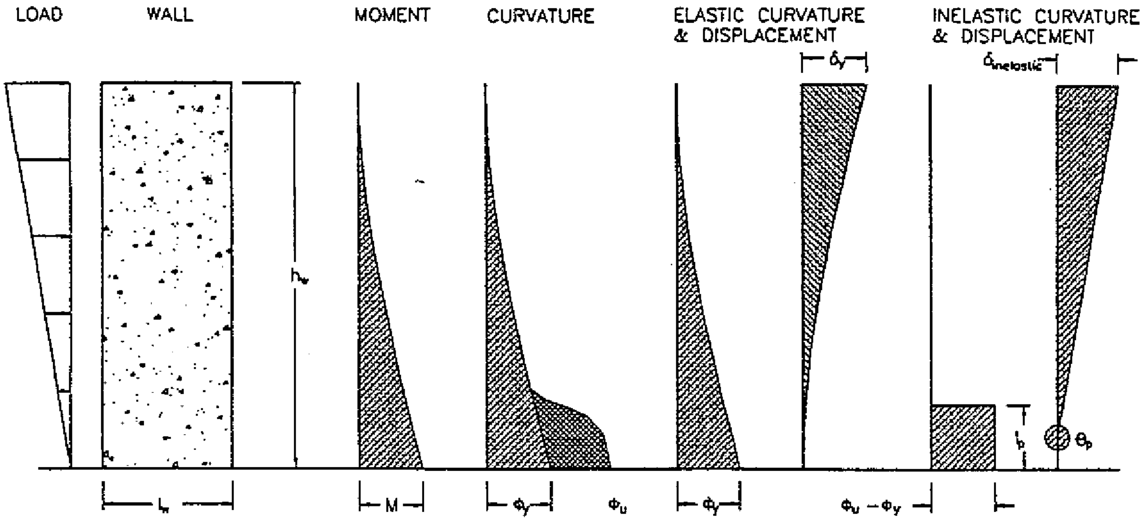


Figure 10 Wall Model and Deformation Components

where f_{yh} is the yield strength of the hoops and cross ties, s is the spacing of the transverse reinforcement, and h_c is the cross-sectional dimension measured center-to-center of the confining reinforcement. The vertical spacing of the hoops and cross ties is limited to the smaller of 6 inches and 6 diameters of the largest vertical bar within the boundary zone. Cross ties or legs of overlapping hoops shall not be spaced further apart than 12 inches along the wall, and alternate vertical bars shall be confined by the corner of a hoop or a cross tie. The ratio of the length to width of a hoop shall not exceed 3 and all adjacent hoops shall be overlapped. The special transverse reinforcement must extend vertically a distance that need not exceed l_w, or $M_u/4V_u$ based on upper-bound estimates of the "plastic hinge length".

The approach presented in UBC-97 may be streamlined, or possible placed in the commentary of ACI 318-2001, to provide greater flexibility to apply alternative procedures that achieve the same result. Commentary will also be added to discuss the need to ensure the critical section occurs at the base of the wall. Additional checks on flexural strength distribution over the height of the wall also may be added to ensure this requirement is satisfied.

Requirements For Flanged Walls: Special requirements were incorporated into the UBC for walls with flanges (walls with I-, T-, L-, and C-shaped cross sections). For UBC-94, the required flange width was limited to half the distance to an adjacent shear wall web, or 15% of the wall height (UBC, 1994, Section 1921.6.5.2). For UBC-97, an the effective flange width must be calculated for both the flange in tension and the flange in compression. This provision was added to address the conflicting issues of flexural strength, the wall shear that develops at the wall flexural strength, and the need for transverse reinforcement at the free end of the wall web (Wallace, 1996). For the flange in compression, the flange width is one-half of the distance to an adjacent wall web or 15% of the wall height. For the flange in tension, the flange width is one-half of the distance to an adjacent wall web or 30% of the wall height.

Although the modifications to UBC-97 attempt to address the conflicting needs noted in the previous paragraph, they do not do so in a clear manner. To ensure a conservative flexural strength estimate for code level forces (reduced by R_w), a lower bound estimate of the effective flange width should be used for both the compression and tension flange width, although it is noted that the value used for the compression flange width does not influence flexural strength measurably. To ensure a conservative design for the shear developed at flexural strength, confinement of the free end of the web, and the potential to fracture the tension steel at the free end (Wood, 1989) an upper bound estimate should be used. The UBC 1997 provisions, as written, do not clearly address this need. A compromise value for the effective flange width, that can be used for analysis for both the flange in tension and compression would be advantageous. This would allow an engineer to use a single wall cross section to assess all design issues (stiffness, strength, and detailing). A review of experimental results (Thomsen and Wallace, 1995) suggests that an effective flange width of eight times the flange thickness on each side of the web, but not exceeding one-half the distance to an adjacent wall web, is a reasonable approximation (for both tension and compression). Alternatively, a value of 20 to 25% of the wall height could be used as a compromise value for both the flange in tension and compression. Commentary will be added to ACI to discuss the important considerations associated with selecting an effective flange width (strength, stiffness, and detailing).

Walls with Openings

Walls with openings can generally be classified as aligned openings (or coupling beams) and non-aligned openings (or random openings). The following items are being considered:

Coupling Beams: Design provisions for coupling beams in UBC-97 are different then those in ACI 318-95. For moderate-to-low aspect ratio $(l_n / d < 4)$ coupling beams with moderate-to-high shear stress $\left(v_u > 4\sqrt{f'_c}\right)$, UBC requires that diagonal reinforcement be used, whereas ACI does not. Use of diagonal reinforcement requires thicker walls and results in significant steel congestion; therefore, use of diagonal reinforcement should be limited to conditions where a measurable improvement in beam behavior is likely. Tests results reported by Paulay (1971) indicate that coupling beams with span-to-depth ratios of 2.0 do not behave substantially different from common reinforced concrete beams. In ACI 318-2001, the use of the 1997 NEHRP provisions, that promote the use of diagonal reinforcing in coupling beams, is being considered. For shear levels exceeding $4\sqrt{f'_c}$ psi $\left(0.33\sqrt{f'_c}\ \text{MPa}\right)$ the lower limit on ratio of clear span to effective depth (l_n/d) may be reduced from four to two based on a review of experimental studies. Alternatives to diagonal reinforcing will also be allowed if it can be shown that the performance is adequate.

Isolated or Random Openings: Use of strut-and-ties models and capacity design will likely be adopted to address walls with random or isolated openings. Research has shown that this approach is effective for both low and high aspect ratio walls (Yañez et al., 1991; Taylor, Cote and Wallace, 1997).

Design for Shear

Slender Walls: The UBC (1994 or 1997)) does not require calculations to estimate the expected wall shear force at the probably flexural strength for slender walls. This could be a significant shortcoming, especially for flanged walls (Wallace, 1995), where the actual wall shear may be several times the code specified shear force. An estimate of maximum shear force expected to develop in the wall should be required to ensure adequate shear strength is provided.

Low-rise Walls: Current code provisions tend to focus on the design and detailing of slender walls; Issues relating to low-rise walls also need to be addressed. Strength and failure modes should be checked using strut and tie models to promote limit states that involve yielding of tension steel (versus crushing or sliding). In addition to reviewing current code equations for shear strength, limits on maximum shear stress are being reviewed. Current provisions allow shear stress as high as $1\ 0\sqrt{f'_c}$ psi $\left(0.83\sqrt{f'_c}\ \text{MPa}\right)$ on individual wall piers. Test results do not support the use of such a high value, especially for walls without well confined boundary zones (Aktan and Bertero, 1985). A more practical limit, on the order of $6\sqrt{f'_c}$ psi $\left(0.5\sqrt{f'_c}\ \text{MPa}\right)$ for walls without well confined boundary zones and possibly as high as $8\sqrt{f'_c}$ psi $(0.67\sqrt{f'_c}$ MPa) for walls with well confined boundary zones may be more appropriate.

New Materials

Although interest in the use of high-performance materials in high seismic regions has increased in recent years, widespread use has not been achieved due to a lack of experimental studies and code provisions. Analytical studies (Wallace, 1997) indicate that there is no substantial benefit from using high-strength concrete for slender reinforced concrete walls. In general, deflections increase for slender HSC walls (with the same flexural strength as moderate-strength walls) because the increase in modulus is not sufficient to compensate for the smaller moment of inertia (due to the smaller cross section). In addition, larger quantities of transverse reinforcement may be needed at the wall boundary to confine the HSC compared with moderate-strength concrete walls. Although benefits are not apparent for slender HSC walls, the increased shear modulus of HSC can be exploited for design of low-rise walls. A review of limited data from experimental studies of HSC walls conducted in Japan by Kabeysawa and Hiraishi (1997) indicates that $9\sqrt{f'_c}A_{cv}$ is a fairly good approximation to wall shear strength for HSC (Wallace, 1997) compared with $6\sqrt{f'_c}A_{cv}$ for moderate-strength walls (Wood, 1990). Kabeyasawa and Hiraishi (1997) recommend use of a combined strut and truss mechanism to compute the shear strength of HSC walls and obtain good correlation between experimental were obtained for this model. It is unlikely that this material will be included in the current code writing efforts until more information is available and it is subject to thorough review.

SUMMARY AND CONCLUSIONS

A review of recent research and code development for ACI 318-2001 was presented and discussed. Code provisions for walls has not kept pace with research and the major codes of practice have different requirements. There is a need to expand the scope of current provisions to address shortcomings and the limitations, incorporate new research findings, and to unify the various design documents.

ACKNOWLEDGMENTS

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